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Stability of Unbound Open-graded Aggregate Layers

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Stability of Unbound Open-graded Aggregate Layers

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Abstract:	The report presents the results of a laboratory study that was carried out to investigate the stability of thin layers of coarse single-sized (open-graded) pavement aggregates under dynamic plate loading. The investigation considered a range of aggregates having variable quality as defined by the Aggregate Index Number. The Aggregate Index Number is a quantitative measure of the combined durability of an aggregate based on its resistance to fracture and abrasion as measured in the Los Angeles Abrasion and Mill Abrasion Tests. Other variables between tests were gradation size and support compressibility. The test results showed that for the coarsest gradation investigated, the semi-logarithmic permanent deformation rate could be reduced significantly by increasing aggregate quality and/or increasing underlying support stiffness. The sensitivity of deformation rate to aggregate quality and underlying support compressibility was observed to decrease for parallel but finer gradations. The results of a test carried out with a 1.5% asphalt cement binder and a test with an unbound aggregate sample that was partially flooded are also presented. Finally, implications to selection criteria for aggregate used in unbound open-graded drainage layers below highway pavements are identified.
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Executive Summary

The report presents the results of a laboratory study that was carried out to investigate the stability of thin layers of coarse single-sized (open-graded) pavement aggregates under dynamic plate loading. Three different aggregate types with three parallel grain size distributions were considered. The two coarsest gradations are considered representative of gradations that have been used as Open-Graded Drainage Layers (OGDL) in highway pavements in North America. The third and finest gradation was used to investigate the influence of relatively small aggregate sizes on base layer stability. The investigation considered a range of aggregates having variable quality as defined by the Aggregate Index Number. The Aggregate Index Number I_a is a quantitative measure of the combined durability of an aggregate based on its resistance to fracture and abrasion as measured in the Los Angeles Abrasion (LAA) and Mill Abrasion (MA) Tests, i.e: $I_a = 5 \times MA + LAA$. Another variable between tests was the compressibility of the underlying support which was either flexible (subgrade modulus = 370 MN/m^3) or rigid. Up to 2 million applications of a 40 kN load using a 460 mm diameter rigid plate were applied to each OGDL sample. The samples were 100 mm thick by 1 m square. The following observations were made:

1. The logarithmic rate of accumulated plastic deformation is dependent on both the aggregate quality defined by the Aggregate Index Number and the compressibility of the underlying support.
2. The logarithmic rate of plastic deformation was observed to diminish with aggregate quality for OGDL samples over a compressible support when constructed with gradations considered representative of current North American specifications.
3. The logarithmic rate of plastic settlement was relatively insensitive to aggregate quality when constructed over a rigid (incompressible) support.
4. The sensitivity of deformation rate to aggregate quality and underlying support compressibility was observed to decrease for parallel but finer gradations.

The results of this study show that if a coarse unbound aggregate gradation (comparable to gradation 1 in this study) is required based on drainage criteria, then an aggregate with a low Aggregate Index Number is needed to ensure adequate resistance to fracturing and abrasion. This is particularly true of OGDL's placed over compliant road support. Under these conditions, the results of the current investigation have shown that a 1000 fold increase in the number of load applications is required for the highest quality aggregate to achieve the same level of permanent deformation as the poorest aggregate. Highway pavement performance is evaluated in part by the magnitude of surface rut depth. It is apparent that if the number of load applications to achieve an unacceptable level of deformation

in the OGDL can be increased by a factor of 1000 then, pavement repair cycle times may be dramatically reduced. An alternative strategy to selecting a better quality material is to use a somewhat finer gradation (comparable to gradation 2 in this study) to improve the stability of the aggregate. The results of a test carried out with a 1.5% asphalt cement binder are also presented.

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1/ Introduction

The Infiltration of surface water into pavement cracks and joints is a major source of pavement deterioration in highway pavements. A strategy that is becoming increasingly more common in North America to reduce or eliminate this problem is to construct an open-graded drainage layer (OGDL) directly below the surface course that can direct infiltrated water to pavement shoulders or drains (e.g. Baldwin and Long 1988, Highlands and Hoffman 1988, Engineering News Record 1979 and Cedergren 1988). An example of a proposed highway pavement structure that incorporates an OGDL is given in Figure 1. Due to the high volume/short duration flows that may be required of an OGDL layer it is necessary that the aggregate have high permeability. A successful OGDL must be capable of removing water at a rate greater than the infiltration rate. For this reason relatively coarse-grained single-sized (open-graded) aggregates are recommended. The aggregate layer must be protected with suitable bases or filters and provided with outlet drains that ensure positive drainage. Due to the proximity of any OGDL to the pavement bearing surface it is necessary that the OGDL possess adequate stability against traffic-induced shearing stresses. These shearing stresses may be further amplified in the vicinity of pavement cracks or, in the case of reinforced concrete pavements, joints. Under these conditions traffic wheel loads may not be attenuated through the surface course and the OGDL may experience overstressing. The necessity of providing both adequate drainage capacity and adequate stability can lead to conflicting requirements in the specification of the aggregate source since the stability of a granular material generally increases if a material has a well-graded particle size distribution (i.e. strength increases with decreasing void ratio). In many instances a 1.5% to 2% asphaltic cement (AC) binder is added to the aggregate to stabilize the layer during construction activities. The AC content is just great enough to coat the aggregate and does not significantly reduce permeability of the material. Nevertheless, unbound open-graded drainage layers have also been successfully constructed as reported by Highlands and Hoffman 1988 and Engineering News Record 1979. The influence of grain size distribution on aggregate permeability in pavement structures has been the topic of investigation by many researchers (e.g. Highlands and Hoffman 1988, Cedergren et al. 1973, Moyanhan and Sternberg 1977, Smith et al. 1965, Strohm et al. 1967). Less well understood is the influence of properties such as fracture resistance (toughness), abrasion resistance (hardness) and grain size distribution of open-graded aggregates on the stability of these materials under repetitive loading (e.g. Winterkorn 1967). Related work by Raymond (1979), Raymond et al. (1979), and Watters et al. (1987) concerned with predicting longevity of ballast aggregates in railway track offers guidance to experimental programs for open-graded aggregates in highway pavements. For example, Canadian Pacific Rail

(CP Rail) has adopted a track model that relates longevity of ballast to aggregate fracture resistance, aggregate hardness and cumulative tonnage on track (CP Rail Specification for Ballast 1984). The work by the second author and others leading to the CP Rail track model has, in part, inspired the experimental approach adopted in the current investigation.

2/ Current Experience at MTO

The current Ministry of Transportation of Ontario (MTO) specifications for OGDL's rely on in-house experience with the performance of traditional aggregates and asphalt mixes that use existing aggregate gradations. Current MTO aggregate specifications and AC content for OGDL construction are given in Appendix A. At the present time pavement structures in Ontario that incorporate an OGDL have not undergone long-term laboratory testing or evaluation. The field performance of Modified HL.4 and HL.8 mixes (that are somewhat similar to OGDL's) shows that the modified mixes are prone to stripping and settlement in the vicinity of transverse cracks. This has been attributed to water penetration, stripping of asphalt cement and the pumping action that may result from traffic loading. It is expected that stripping and settlement of the OGDL can occur with time due to the thin asphalt coating and low initial void ratios achieved during construction. This may occur particularly at the pavement joints and cracks where water penetration is increased and the potential for pumping is largest. An important conclusion that can be drawn from these observations is that the stability of an OGDL in the vicinity of pavement cracking is ultimately controlled by the inherent stability of the stripped aggregate. Based on this observation and discussions with MTO personnel during the initial portion of the work this study has focused on the behaviour of unbound aggregate systems.

3/ Scope Of Current Investigation

This report describes a laboratory investigation that was carried out to examine the influence of aggregate properties on the stability of unbound open-graded drainage layer samples under simulated traffic loading. The properties of the aggregates that were investigated were fracture resistance, hardness and grain size distribution. It should be noted that the scope of the work has been narrowed and refined over the course of the research program from the original proposal as a result of many discussions between the investigators and MTO personnel. Consequently, the current research has not considered the cost of the aggregate, any potential to degradation through weathering nor the AC stripping potential of the aggregates. Finally, the program of work reported here has been restricted to examination of the behaviour of single-sized relatively coarse gradations. These gradations are considered to have such a high permeability that even samples with significant breakdown are expected to retain adequate permeability to meet the drainage function of an OGDL.

4/ Test Program

4.1/ General

Three different aggregates from eastern Ontario were examined. The properties that were used to characterize the aggregates were:

- aggregate gradation (i.e. ranging from a gradation with a top size of 37.5 mm to parallel finer grain size distributions with a top size of 19 mm and 9.5 mm).
- the combination of hardness and resistance to fracturing of the aggregate samples as defined by the Mill Abrasion value, Los Angeles Abrasion value and the Aggregate Index Number.

The relative stability of the aggregate materials was determined by comparing the cumulative load–deformation response of thin layers of aggregate during repeated loading in a laboratory plate loading apparatus. The principal variable between sample configurations with respect to the plate bearing test was the use of either a rigid or flexible support to model a range of compressibility in the underlying road structure. Experience from similar test programs on the stability of railway ballast has shown that the compressibility of the underlying structure has a very important influence on the cumulative load–deformation response of aggregate materials (Raymond and Bathurst 1987). The properties of the aggregates used in the study are given in Table 1 and a summary of test configurations is given in Table 2. The report is focussed on the results of 23 plate bearing tests that were carried out on three principal aggregate types (Tests 6 through 28). Of these 23 tests, four were replicate tests to examine test repeatability. In addition, five preliminary tests were carried out during the development of the test procedure. These tests used unbound and bound samples comprising a crushed limestone aggregate conforming to a HL.8 gradation.

4.2/ Aggregate Sources

The aggregate sources used in this investigation comprised 100% crushed rock. The meaning and significance of the characteristic properties of the aggregates are given in the following sections. The Elginburg Limestone and Jasper Dolomite have been used as base materials in roadworks by MTO in eastern Ontario and the Marmora Trap as ballast in branchline track by CP Rail.

4.3/ Gradation

The standard grain size curves investigated are given in Figure 2. Gradation 1 represents the coarsest material considered and is close to the current OGDL specification presently adopted by MTO (Figure 2b). Gradation 2 (medium) and 3 (fine) have similar slopes and shape to gradation 1 (i.e. similar Coefficient of Uniformity (C_u) and Coefficient of Curva-

ture (C_c)) but are shifted to essentially parallel gradations with finer particle sizes. For comparison purposes the hatched regions in Figure 2c and 2d correspond to gradation specifications that have been used for OGDL construction in the United States. Gradation 1 is close to the AASHTO 57 specification for coarse aggregate (Baldwin and Long 1988) and gradations 1 and 2 fall within specifications for OGDL's recently adopted by the Pennsylvania Department of Transport (PennDOT) (Highlands and Hoffman 1988). The target gradations were achieved by blending pre-sieved aggregate size ranges taken from laboratory stock piles.

4.4/ Los Angeles Abrasion Test

Los Angeles Abrasion (LAA) tests were carried out on aggregate samples as given in ASTM Designations C 535-81 and C 131-81. The LAA tests were carried out to give a relative measure of particle resistance to fracturing. The LAA test is considered to simulate the effect of high contact forces on aggregate particles including those generated by impact loading. High impact forces on OGDL aggregates will occur in the vicinity of transverse cracks in asphaltic pavements and below joints in concrete pavements. The ability of an aggregate to survive high contact forces is dependent on the toughness of the aggregate (i.e. particle resistance to fracturing). A low LAA value indicates a material with a high resistance to fracturing under high contact forces. Since the samples tested covered a range of aggregate sizes the Standard Test Method ASTM C 535-81 was used for materials having a particle size greater than 19 mm and the test method given in ASTM C 131-81 was used for materials with a particle size less than 19 mm.

4.5/ Mill Abrasion Test

The Mill Abrasion (MA) test is a non-standard test that measures the relative resistance to abrasion or hardness of aggregate materials as a result of the autogenous grinding of the aggregate particles. Abrasion of aggregate particles in an OGDL will result from the constant movement of aggregate particles against each other, particularly in the vicinity of pavement cracks and joints. The abrasion mechanism is simulated by the autogenous grinding process in the Mill Abrasion test. The specifications for the MA test are given in Appendix B of this report and are reproduced from the current CP Rail specification (CP Rail Specification for Ballast 1984). In the current investigation the CP Rail method of test was modified for samples having a smaller top-size than that designated in the original specification.

4.6/ Hardness-Toughness Trade-Offs and Aggregate Index Number

Experience with railway ballasts (Raymond 1979, 1987, Raymond et al. 1979) has shown that aggregates that are tough with respect to fracture resistance are not necessarily highly

abrasion resistant. Physical measurements from LAA and MA tests (Watters et al. 1987, Boucher and Selig 1987) in support of this hypothesis can be seen in Figure 3. Similar scatter in the data for other aggregates has been reported by Raymond (1987). The practical consequence of this phenomenon to railway applications is that the same longevity of ballast under cumulative tonnage can be obtained by aggregates having different combinations of MA (hardness) and LAA (toughness) values. A parameter that reflects the contribution of both mechanisms to aggregate durability in track is the Aggregate Index Number (I_a) where:

$$I_a = 5 \times MA + LAA$$

The results of repeated load (large diameter) triaxial tests carried out on a variety of railway ballasts (Raymond and Diyaljee 1979) has shown that the logarithmic rate of accumulated plastic strain in these assemblies is strongly correlated to the Aggregate Index Number as illustrated in Figure 4. CP Rail has adopted limits on the Aggregate Index Number as a ballast criterion and have correlated the I_a value and aggregate gradation with ballast life (i.e. cumulative tonnage to aggregate fouling). In the CP Rail specification the Aggregate Index Number is called the Abrasion Number. The work by Raymond and CP Rail has inspired the current experimental approach and has led us to investigate Aggregate Index Number as an indicator of relative stability of coarse single-sized highway aggregates. The aggregates that were tested in the current investigation were selected to give a range of hardness and fracture resistance and Aggregate Index Number. The Elginburg Limestone was the poorest quality aggregate investigated (i.e. greatest I_a value) and in qualitative terms represents an aggregate with low hardness/low toughness (soft/weak). The Marmora Trap is a metamorphic rock and was the highest quality aggregate (i.e. lowest I_a value) investigated. It can be said to have medium hardness/high toughness. The Marmora Trap aggregate has been stockpiled in the outdoors at RMC for a number of years and has been investigated by the authors as a potential ballast source in track (Raymond 1986). It is of interest to note that this material is susceptible to weathering as a result of chlorite minerals present in the rock matrix. The susceptibility to weathering has been traced by carrying out MA tests at intervals. When freshly quarried and crushed this material recorded a MA value ranging from three to four. The current value from Table 1 is about 5.1. The Los Angeles Abrasion value of 14.1 has remained unchanged. Consequently, the Aggregate Index Number for this material has increased from about 29 to 40 due to weathering. It is reasonable to assume that this particular aggregate would not be recommended in an actual field application due to its susceptibility to weathering. The comments made here highlight the need to consider the susceptibility of aggregate to weathering in the selection procedure

for OGDL materials. Nevertheless, the Marmora Trap was employed in this investigation to ensure a wide range of aggregate performance in terms of mechanical stability. Finally, the Jasper Dolomite is an intermediate quality aggregate defined by medium hardness/medium toughness as determined from the results of MA and LAA tests.

4.7/ Plate Bearing Test Procedure

The general arrangement for the plate bearing test is given in Figure 5. An overview of the test system is shown in Figure 6. The OGDL is modelled as a layer of aggregate 100 mm thick and is supported by a rigid or flexible base. The 100 mm thickness is typical of OGDL installations in Ontario and in the United States (Baldwin and Long 1988, Ministry of Transportation of Ontario communication). Nevertheless, 100 mm is likely the minimum thickness that would be considered in actual installations in order to facilitate construction and to ensure adequate flow capacity. The OGDL samples were one meter by one meter in plan area and confined within a rigid thick-walled aluminum tray. The tray permits unbound aggregate samples to be placed and compacted away from the loading system. The flexible support condition was created by using a closed cell gum-rubber mat (12 mm thick). The equivalent subgrade modulus (k_s) of the mat material was determined to be 370 MN/m^3 using a rigid 457 mm diameter steel plate and a static load of 40 kN (i.e. maximum bearing pressure = 244 kPa). The measured mat flexibility is considered to be representative of the compliance due to the combined effect of a good granular subbase overlying a competent cohesive subgrade. In a previous investigation (Raymond and Bathurst 1987) the CBR of this artificial 'subgrade' was estimated to be equivalent to a granular material with a CBR of 40. A sinusoidal loading pulse with an amplitude of 2 to 40 kN was applied to the surface of each sample using the same rigid steel plate described above. The maximum 40 kN load represents a standard half-axle load. The 457 mm diameter plate (as opposed to say a 300 mm diameter plate) was selected in order to simulate the increased bearing area that would exist in the pavement structure at the elevation of the OGDL as a result of vertical load spreading through the overlying surface course. The minimum load level of 2 kN was selected to ensure that there was a good contact between the aggregate and the loading plate at all times. The load was delivered to the plate using an MTS closed-loop electro-hydraulic actuator controlled by a DEC PDP11/34 computer. A load cell and linear variable displacement transducer (LVDT) located above the actuator base were used to monitor footing loads and vertical displacements. At programmed intervals, the load-deformation response of the footing during a loading cycle was recorded and stored by the computer. The load was applied at a rate of 5 Hz for one to two million load applications. The plate was maintained in a horizontal position throughout the test by passing the actuator

piston through a rigidly-supported Teflon bushing (Figure 7). Also indicated on the general test arrangement in Figure 5 is a concrete plinth that was used to support the aluminum tray confining the aggregate sample. The purpose of this structure is to provide a catchment system for in-situ permeability tests. The results of permeability tests are described later in the report. The aggregate samples were compacted using a hand-held tamping plate weighing 10.4 kg and having a tamping area 250 mm by 250 mm. The same number of passes and height of fall were used in the preparation of each specimen.

4.8/ Aggregate Density

For comparison purposes a series of maximum and minimum density tests were carried out on unbound samples of Pittsburgh Limestone, Elginburg Limestone, Jasper Dolomite and Marmora Trap conforming to HL.8 Modified and trial gradations 1, 2 and 3. The maximum densities achievable were determined using a vibratory table and a method of test developed at Queen's (Gaskin et al. 1978). The test was carried out using a large diameter 250 mm brass mold 200 mm high bolted to the top of a vibratory table. Each sample was placed in a loose condition and then subjected to a surcharge pressure of 70 kPa. Vertical accelerations of about 1.6g were then applied to the sample by the vibratory table while maintaining the surcharge pressure at 70 kPa. Test results were found to be independent of the magnitude of the surcharge pressure over the range of 0 to 150 kPa investigated. A principle benefit of this test arrangement was that maximum sample densities were achieved without particle fracturing. Compaction of samples using a tamper method would have had the disadvantage that individual aggregate particles would be broken and the gradation altered (as would be the case if the samples were prepared according to specifications for Proctor densities). Minimum densities were determined according to the methods described in ASTM D 4254-83.

5/ Test Results

5.1/ Results of Density Tests on Unbound Aggregates

The results of density measurements from as-constructed plate bearing test samples, vibratory table tests and minimum density tests are summarized in Tables 3 and 4. The data shows that for the as-compacted samples of Elginburg Limestone, Jasper Dolomite and Marmora Trap the void ratios vary from about 0.42 to 0.61. The same data is plotted in Figure 8a and it can be seen that there is no systematic variation in void ratio between samples constructed with a rigid support. The void ratios for the flexible support models with the finest gradation are somewhat lower than for the comparable rigid support configurations. Nevertheless, the mean value of void ratio at a particular gradation is reasonably constant at about 0.49 for both rigid and flexible models. As-constructed relative densities for plate-bearing plate samples are plotted in Figure 8b. The calculated relative densities from all tests gave a mean value of about 0.68 which is also a reasonable value for all aggregate types and gradations for samples constructed with a rigid support. The spread in data about this mean is, however, greater for the flexible base samples, particularly those constructed with the finest aggregate sizes (gradation 3). As part of on-going research for Phase II of the current project, bound OGDL samples have been recently cored from several Ontario Highways. The densities of limestone samples meeting HL.8 Modified specifications gave a range of densities from 1.79 to 1.89 Mg/m^3 with a mean of 1.85 Mg/m^3. A reasonable estimate of the void ratio for these aggregates in-situ is $e = 0.45$ which compares favourably to the values achieved for unbound Elginburg Limestone samples compacted in the plate bearing test apparatus.

5.2/ Performance of Unbound Aggregates under Repeated Loading

The results of plate bearing tests with the coarsest aggregate (gradation 1) are shown in Figures 9 and 10. Figures 9a and 10a show that the cumulative (permanent) displacement with number of load repetitions is highly non-linear with most of the deformation occurring early in the loading program. The same data are replotted with semi-logarithmic axes in Figures 9b and 10b. In general, the data show that over a wide range of load repetitions the permanent displacements vary linearly with the log number of load applications. The data from these tests indicates that the magnitude of permanent displacement increases with the value of Aggregate Index Number I_a and with the compressibility of the underlying support. Permanent deformation as a relative measure of aggregate stability under load can be misleading since the magnitude of permanent deformation after a given number of load applications is sensitive to the effects of initial plate seating. In this investigation the most reliable measure of relative stability between tests was found to be the rate at

which permanent deformation is accumulated with respect to the log number of load repetitions (i.e. mm/log cycle) taken over 10 to 10^6 load applications. For brevity in the following text this quantity is referred to as deformation rate. Deformation rates versus aggregate size for all test configurations are summarized on Figures 11b,c,d based on the three principal aggregates investigated. For the poorest aggregate (Elginburg Limestone in Figure 11b), the test results show that deformation rates diminish with finer aggregate size and lower support stiffness. Figure 11c shows that for the intermediate quality aggregate (Jasper Dolomite) the deformation rates are essentially insensitive to aggregate size but decrease with lower support compressibility. The same data for the highest quality aggregate (Marmora Trap) shows that deformation rates are essentially independent of aggregate size and support condition (Figure 11d). The influence of test parameters on sample stiffness can be described by rebound values measured over the course of each test. The rebound values have been calculated as the difference between permanent and peak plate deformations during a load cycle. An average rebound value was calculated for 10 to 10^6 load applications in each test and these values are plotted in Figures 12a and 12b. These figures illustrate that rebound values are essentially insensitive to grain size but (as expected) are dependent on the support stiffness. For the flexible support condition, the rebound values show a tendency to increase with decreasing aggregate quality. Nevertheless, it should be noted that the magnitude of the values recorded are close to the limit of the instrumentation accuracy and it is therefore difficult to confidently isolate the influence of aggregate quality on system rebound values. The deformation rate values from all tests can be replotted against Aggregate Index Number (I_a) as shown in Figures 13b, 13c and 13d for gradations 1 (coarse), 2 (medium) and 3 (fine) respectively. The figures illustrate that the stability of OGDL's meeting the coarsest size specification can be improved by selecting a higher quality aggregate as defined by its Aggregate Index Number. This is particularly true for open-graded aggregates over compressible bases. For example, reducing the Aggregate Index Number from 74 to 40 corresponds to a reduction in the (log) cumulative deformation rate by a factor of three. On a linear scale this improvement corresponds to a 1000 fold increase in the number of load applications to achieve the same deformation level as for the poorest aggregate. Figure 13c illustrates that the performance of OGDL's having a size distribution corresponding to a medium gradation (gradation 2) can only be improved by selecting a higher quality aggregate if the layer is over a compressible base. Finally, for the finest gradations investigated, Figure 13d shows that the quality of the aggregate does not systematically influence the stability of these systems based on deformation rate.

5.3/ Additional Tests

5.3.1/ OGDL with AC Binder

An original objective of the current test program was to test both unbound OGDL specimens and similar specimens bound with a 1.5% AC binder (85–100). A great deal of effort was expended to devise a method of manufacturing the very large volumes of hot mix required for each test specimen (i.e. approx. 180 kg). After a number of trials a technique was developed to manufacture large samples using a BOMAG hot drum mixer. The mixer is on indefinite loan from the City of Kingston and allows us to manufacture a test specimen in one mix. The mix components are prepared according to standard procedures in our asphalt laboratory. The liquid AC is added to the pre-heated aggregate in the BOMAG mixer. After thorough mixing the hot mix is dumped into the specimen tray (Figures 5 and 6) and compacted with a hand tamper as it cools. The specimen is then transported to the plate bearing test apparatus for repeated load testing and in-situ permeability testing if required. Plate bearing tests were carried out on bound and unbound Pittsburgh limestone specimens having the gradation shown in Figure 14. The test results are plotted in Figure 15. The data shows that the permanent and peak settlement values for both specimens after one million load applications are similar. However, the unbound material gave a lower rate of accumulated settlement after the first few load applications compared to the bitumen sample despite a lower initial density (1690 kg/m^3 versus 1740 kg/m^3 for the bound sample). It should be emphasized that the data in Figure 15 is for two tests only and these tests were undertaken during the development of the test apparatus. In addition, the tests involved a relatively poor quality limestone and it is not known what influence a more durable aggregate may have on similar test results. Recent work as part of Phase II of the research on OGDL stability has involved recovery of intact OGDL cores from actual pavement structures. Densities determined for these samples using ASTM D 1188-83 method of test gave values of about 1850 kg/m^3 as reported earlier. It appears that in the one trial experiment with bitumen bound aggregate the compacted density was low and this may also explain in part the relative performance difference between the bound and unbound aggregate samples. Nevertheless, the data does show that while the bound single-sized aggregate may be easier to place and roll, more compactive effort may be required to achieve the same initial density. If comparable densities to those of unbound specimens are not achieved during the construction phase then a greater level of permanent deformation may occur after construction is complete.

5.3.2/ In-situ Permeability Testing

An original objective of the research program was to develop a method of carrying out in-situ permeability testing of OGDL samples. The test apparatus that was developed to meet this objective is shown in Figure 16. The principal component is a circular steel collar that can be inserted between the surface of the OGDL and the loading plate at any stage during repeated loading. The collar is inserted by lifting the loading plate using the MTS actuator. The collar is then made water-tight by placing a small load on the bearing plate and rubber gasket shown in the figure. The actuator load also compresses the rubber mat directly on the aggregate surface and in this way forces water entering the collar to flow radially through the aggregate. The infiltrated water is then able to drain through the perforated sidewalls of the sample tray. The vent shown attached to the collar ensures that the head of water directly over the aggregate is not more than about 25 mm. The rate of flow achievable with this system is currently up to 6 cfm (2800 cc/sec). Equivalent permeabilities for bound and unbound aggregate specimens using the gradation shown in Figure 14 were calculated to be in excess of 5600 ft/day (2 cm/sec). After discussions with MTO personnel it was agreed that permeabilities of any of the gradations under study would not be expected to fall below an acceptable level of (say) 1000-3000 ft/day. Consequently, permeability aspects of the performance of OGDL in this research program have been discontinued for the time being.

5.3.3/ Flooded Test

A single test (Test 14) was carried out on an OGDL specimen constructed with Elginburg limestone conforming to gradation 1. The results of the test showed that for this coarse gradation there was no appreciable difference in load-deformation response when compared with the equivalent dry sample and this test has been included in the test data for the unbound samples reported earlier.

6/ Conclusions

The results of the current laboratory investigation have shown preliminary qualitative relationships between aggregate stability under dynamic loading and the quality of the aggregate and the stiffness of the underlying support. The coarse and medium-sized open-graded aggregates investigated cover a range that would be considered in OGDL construction based on a number of current specifications that ensure adequate flow capacity for these aggregates when used as drainage layers below pavement. The tests show that the ability of the unbound aggregate to resist cumulative permanent deformation generally diminishes as the quality of the aggregate decreases (i.e. increasing Aggregate Index Number) and as the compressibility of the underlying support increases. For example, if a coarse aggregate gradation (say gradation 1) is desirable based on drainage criteria then an aggregate with a low Aggregate Index Number is needed to ensure adequate resistance to fracturing and abrasion. This is particularly true of OGDL's placed over compliant road support. Under these conditions, the results of the current investigation have shown that a 1000 fold increase in the number of load applications is required for the highest quality aggregate to achieve the same level of permanent deformation as the poorest aggregate. Highway pavement performance is evaluated in part by the magnitude of surface rut depth. It is apparent that if the number of load applications to achieve an unacceptable level of deformation in the OGDL can be increased by a factor of 1000 then pavement repair cycle times may be dramatically reduced. An alternative strategy to selecting a better quality material is to use a finer gradation (say gradation 2) to improve the stability of the aggregate. It should be noted that it is current practice in many jurisdictions to use a 1.5% to 2% AC binder to stabilize the open-graded drainage layer during construction. The influence of parameters such as grain size distribution, aggregate quality, support compressibility, AC content and compaction density on these bound systems warrant investigation. Nevertheless, it has been observed in some MTO highway test sections by the authors that stripping of the OGDL aggregate can occur due to water infiltration and therefore it can be argued that ultimately the stability of the OGDL is dependent on the unbound condition of the aggregate. In practice the Aggregate Index Number should not be considered a substitute for petrographic evaluation of highway aggregates. Petrographic analysis is a primary tool in aggregate evaluation particularly with respect to mineral composition (e.g. minerals may be susceptible to weathering), consistency, particle shape and structure (e.g. foliation, cleavage and bedding planes). A thorough petrographic evaluation should be carried out first to eliminate aggregates that are unsuitable regardless of their Aggregate Index Number. Nevertheless, as this investigation has shown, the interrelation between Aggregate Index Number, gradation size and support compressibility does reveal important trends that can

assist the engineer to choose between aggregates that are potential candidates for OGDL construction.

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Table 1

Summary of Aggregate Properties

Aggregate Type	Specific Gravity	Mill Abrasion (MA) (%)	Los Angeles Abrasion (LAA) (%)	I_a†	C_u††	C_c‡
Pittsburgh Limestone (HL.8 Modified)	2.68*	7.1	22.0	57	2.5	1.1
Elginburg Limestone	2.68	9.1	28.9	74	3.6 - 4.3	1.0 - 1.3
Jasper Dolomite	2.76	6.6	20.9	54	3.9	1.1
Marmora Trap	2.98	5.1	14.1	40	4.0 - 4.4	1.0 - 1.3

† Aggregate Index Number $I_a = 5 \times MA + LAA$
†† Coefficient of Uniformity D_{60}/D_{10}
‡ Coefficient of Curvature $D_{30}^2/D_{60}D_{10}$
* assumed

Table 2

SUMMARY OF PLATE BEARING TESTS
MTO PROJECT 1988/89
STABILITY OF OPEN-GRADED DRAINAGE LAYERS (PHASE I)
PLATE DIAMETER = 457 mm
LOAD AMPLITUDE = 12 TO 244 kPa

Test No.	Aggregate Type	Gradation Type	Support Condition	Load Applications	Date	Remarks (see Notes)
1	L-Pitt	HL.8	Rigid	1000000	21-28/4/88	(1)
2	L-Pitt	HL.8	Rigid	300000	12-17/5/88	(2)
3	L-Pitt	HL.8	Rigid	1000000	18-24/5/88	(3)
4	L-Pitt	HL.8	Flexible	2003000	15-20/6/88	(3)
5	L-Pitt	HL.8	Rigid	1000000	20-22/6/88	(3)
6	L-Elgb	1	Rigid	1000000	24-26/8/88	
7	L-Elgb	1	Flexible	700000	27-29/6/88	
8	L-Elgb	1	Flexible	1200000	08-11/7/88	repeat test
9	L-Elgb	2	Rigid	2000000	30/6/-5/7/88	
10	L-Elgb	2	Flexible	1000000	05-07/7/88	
11	L-Elgb	2	Flexible	2700000	27/7/-2/8/88	repeat test
12	L-Elgb	3	Rigid	2000000	13-18/7/88	
13	L-Elgb	3	Flexible	1000000	18-20/7/88	
14	L-Elgb	1	Rigid	2000000	19-23/8/88	flooded
15	T-Marm	1	Rigid	2500000	21-27/9/88	
16	T-Marm	1	Flexible	2500000	27/9/-3/10/88	
17	T-Marm	2	Rigid	2500000	4-11/10/88	
18	T-Marm	2	Flexible	2500000	11-17/10/88	
19	T-Marm	3	Rigid	2500000	19-31/10/88	
20	T-Marm	3	Flexible	2500000	4-16/10/88	
21	D-Jasp	1	Rigid	2500000	10-16/1/89	
22	D-Jasp	1	Rigid	2500000	02-08/2/89	repeat test
23	D-Jasp	1	Flexible	2501000	08-14/2/89	
24	D-Jasp	2	Rigid	2500000	01-07/3/89	
25	D-Jasp	2	Flexible	2500000	07-13/3/89	
26	D-Jasp	3	Rigid	2540000	03-09/4/89	
27	D-Jasp	3	Flexible	2870000	11-18/4/89	
28	T-Marm	1	Rigid	3610000	26/5/-5/6/89	repeat test

NOTES:

(1) gradation at source, 1.5% A.C. binder, permeability test
(2) gradation at source, unbound, permeability test
(3) gradation at source, unbound

L-Pitt = Pittsburgh Limestone
L-Elgb = Elginburg Limestone
T-Marm = Marmora Trap
D-Jasp = Jasper Dolomite

Table 3

Summary of Aggregate Density Testing
RIGID Support Condition

Aggregate Type	Gradation Type†	Max.†† Density (Mg/m^3)	Min.†† Density (Mg/m^3)	Insitu Density (Mg/m^3)	Max.†† Void Ratio (e)	Min.†† Void Ratio (e)	Insitu Void Ratio (e)	Relative Density‡ D_d
Pittsburgh Limestone	HL.8 Mod.	1.90	1.46	1.69	0.84	0.41	0.59	0.58
Elginburg Limestone	1	2.14	1.40	1.79	0.91	0.26	0.50	0.63
	2	2.13	1.47	1.87	0.83	0.26	0.44	0.68
	3	2.08	1.50	1.81	0.79	0.29	0.48	0.62
Jasper Dolomite	1	1.96	1.53	1.81	0.81	0.41	0.52	0.72
	2	2.14	1.53	1.89	0.80	0.29	0.46	0.67
	3	1.98	1.57	1.89	0.76	0.39	0.46	0.81
Marmora Trap	1	2.15	1.59	1.96	0.88	0.39	0.52	0.73
	2	2.26	1.68	2.03	0.77	0.32	0.47	0.66
	3	2.11	1.62	1.95	0.84	0.41	0.53	0.72

† three maximum and minimum density tests per gradation type
†† from results of vibratory table test and minimum density method of test (ASTM D 4254-83)
‡ $D_d = (e_{max} - e)/(e_{max} - e_{min})$

Table 4

Summary of Aggregate Density Testing
FLEXIBLE Support Condition

Aggregate Type	Gradation Type†	Max.†† Density (Mg/m^3)	Min.†† Density (Mg/m^3)	Insitu Density (Mg/m^3)	Max.†† Void Ratio (e)	Min.†† Void Ratio (e)	Insitu Void Ratio (e)	Relative Density‡ D_d
Elginburg Limestone	1	2.14	1.40	1.81	0.91	0.26	0.48	0.66
	2	2.13	1.47	1.82	0.83	0.26	0.48	0.61
	3	2.08	1.50	1.89	0.79	0.29	0.42	0.74
Jasper Dolomite	1	1.96	1.53	1.84	0.81	0.41	0.50	0.77
	2	2.14	1.53	1.90	0.80	0.29	0.45	0.69
	3	1.98	1.57	1.90	0.76	0.39	0.45	0.84
Marmora Trap	1	2.15	1.59	1.95	0.88	0.39	0.53	0.70
	2	2.26	1.68	1.96	0.77	0.32	0.52	0.55
	3	2.11	1.62	1.85	0.84	0.41	0.61	0.53

† three maximum and minimum density tests per gradation type
†† from results of vibratory table test and minimum density method of test (ASTM D 4254-83)
‡ $D_d = (e_{max} - e)/(e_{max} - e_{min})$

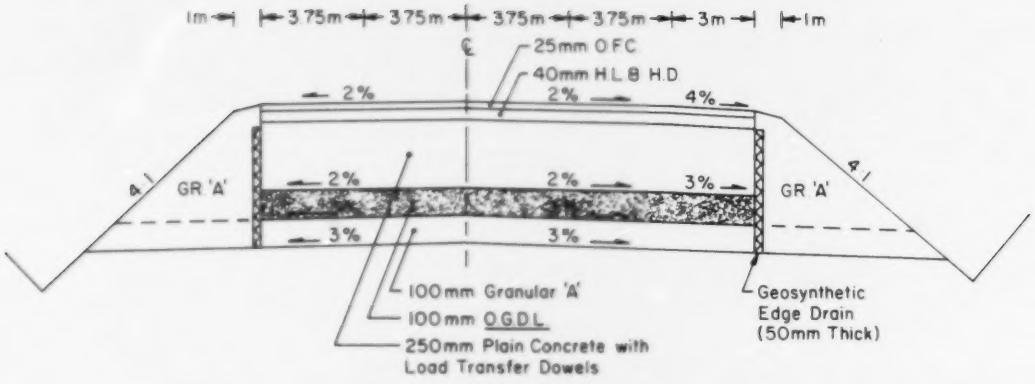
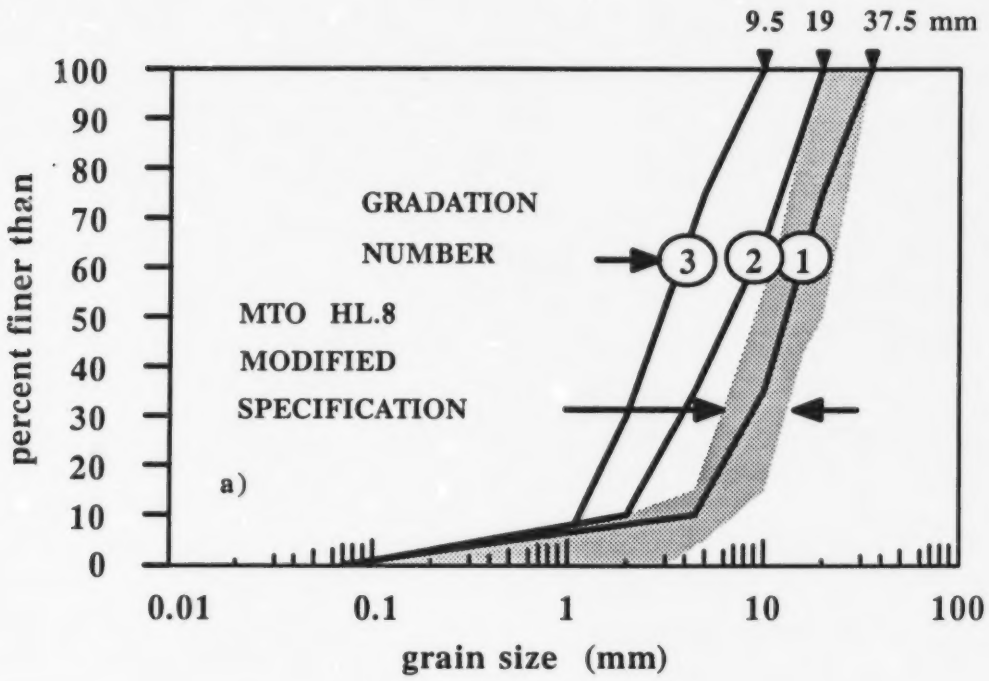


Figure 1/ Example of Highway Pavement with Open-Graded Drainage Layer (proposed section for HWY 407)



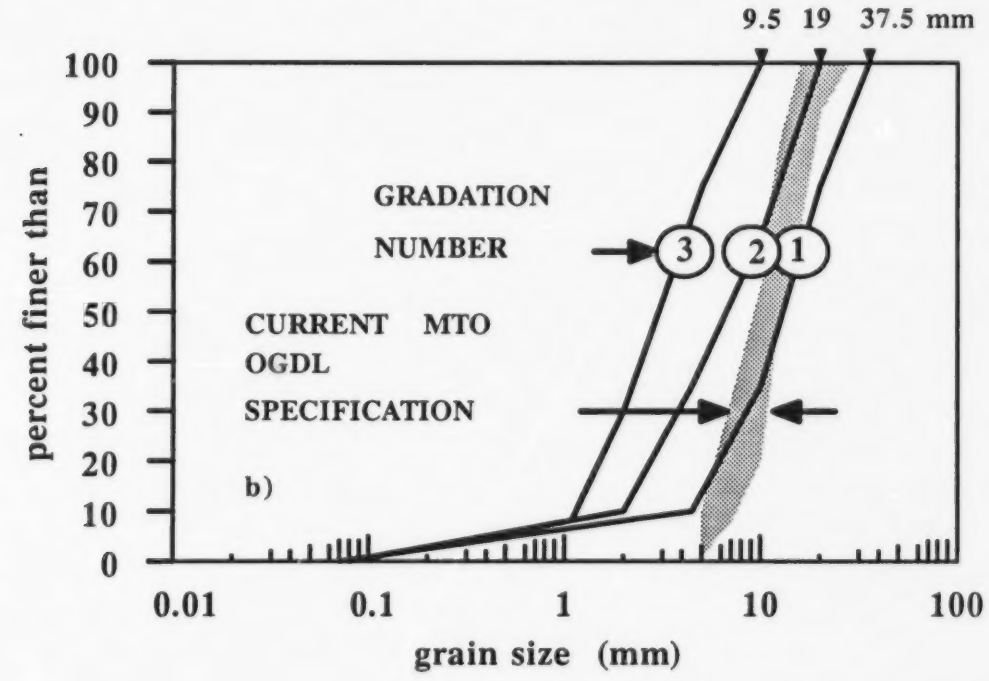
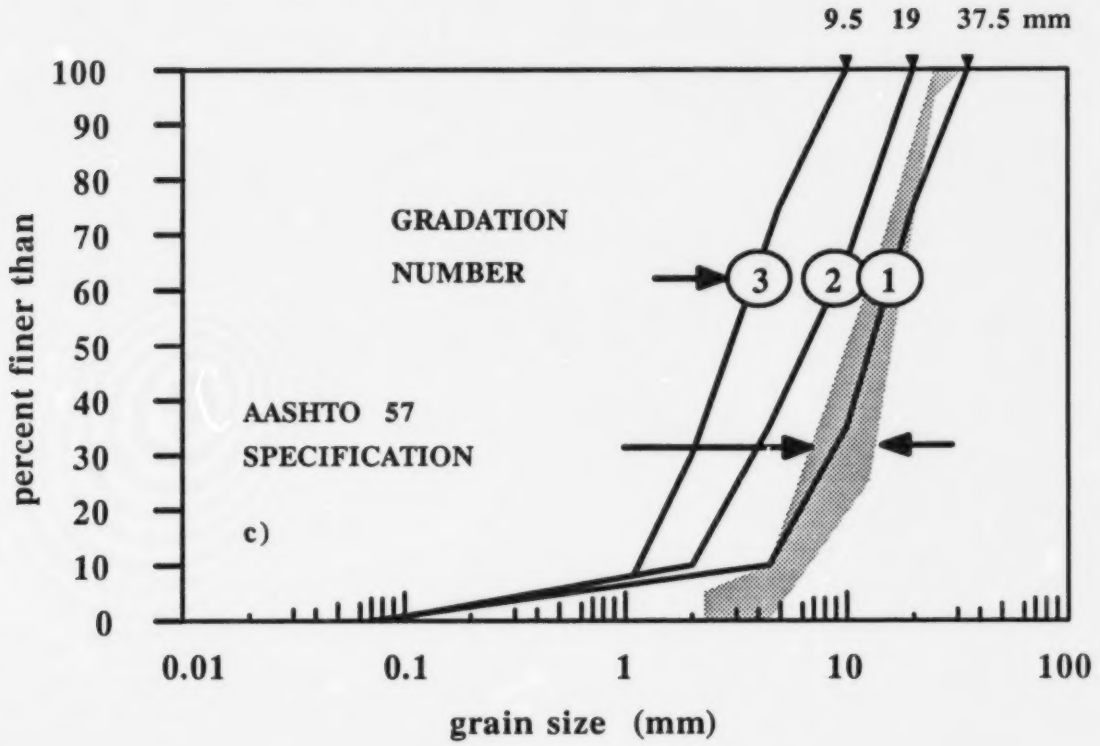


Figure 2/ Examples of Grain Size Distribution for OGDL Aggregates



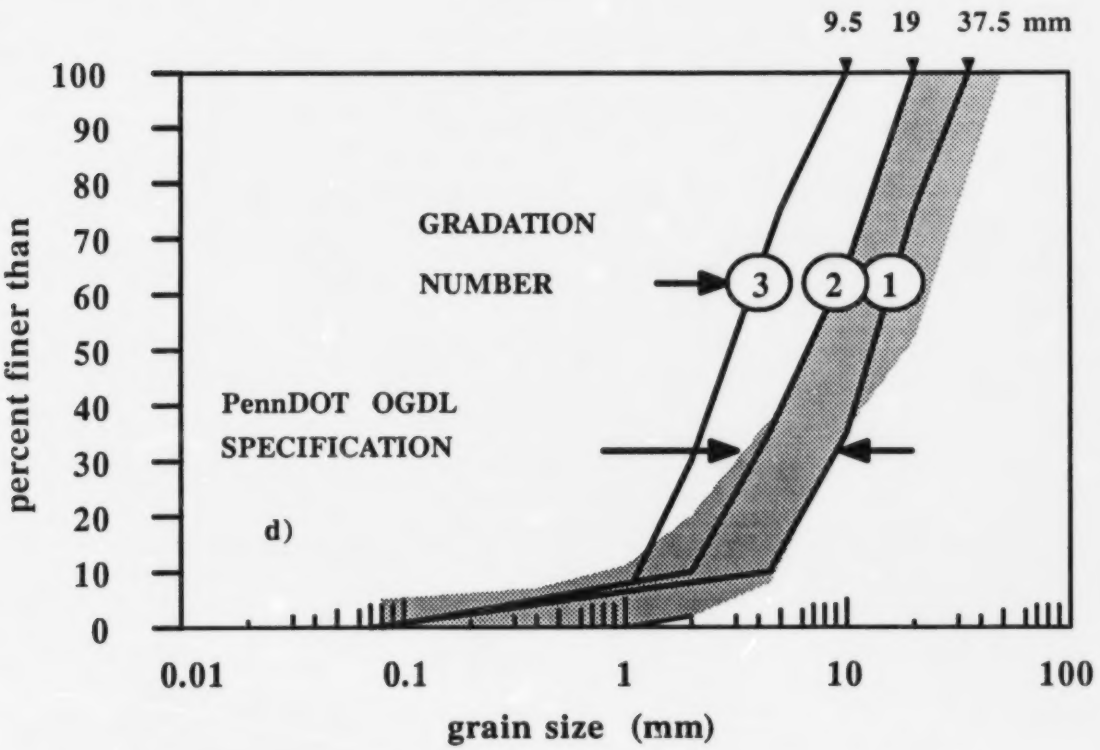


Figure 2/ (cont'd) Examples of Grain Size Distribution for OGDL Aggregates

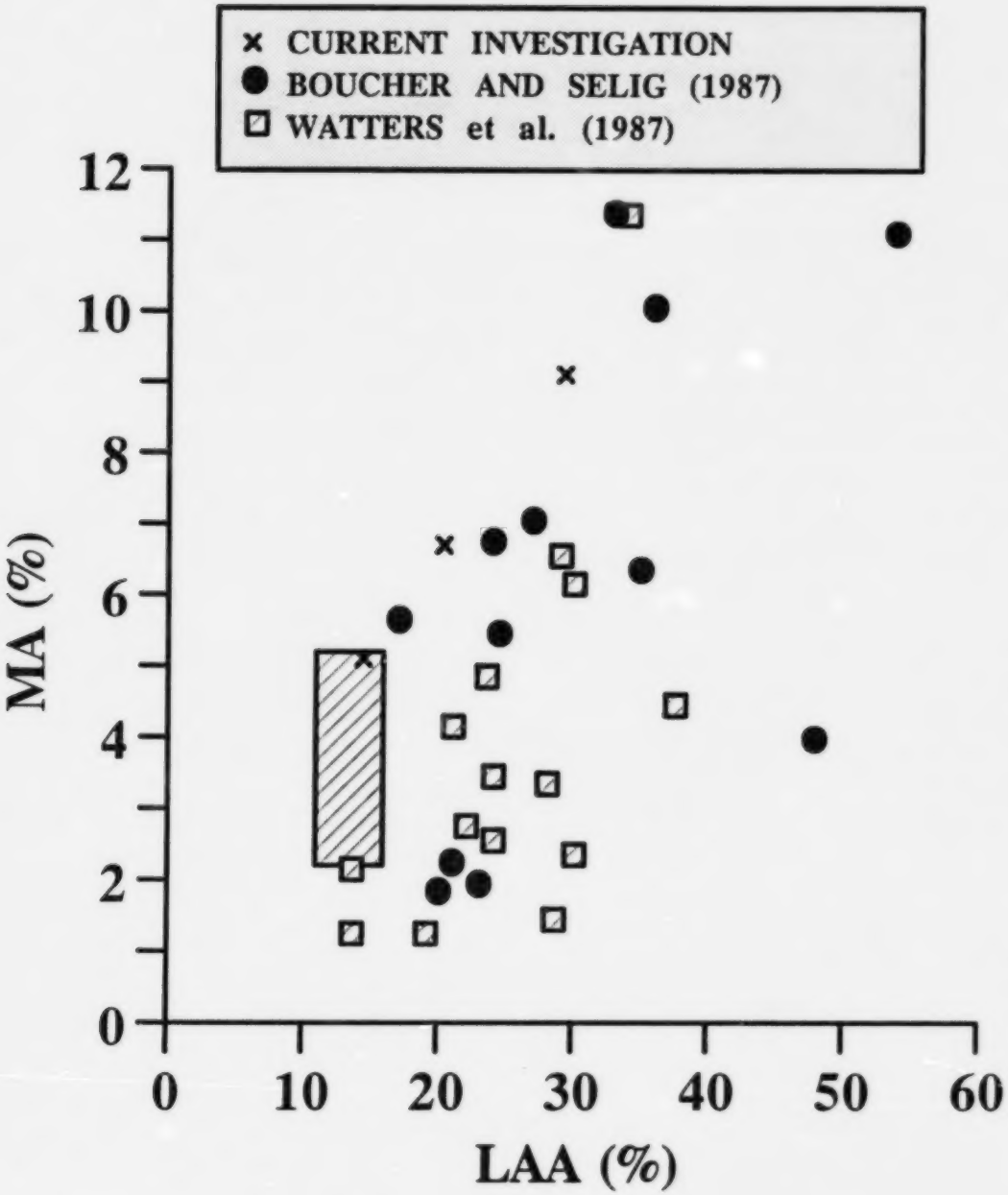


Figure 3/ Mill Abrasion versus Los Angeles Abrasion Values

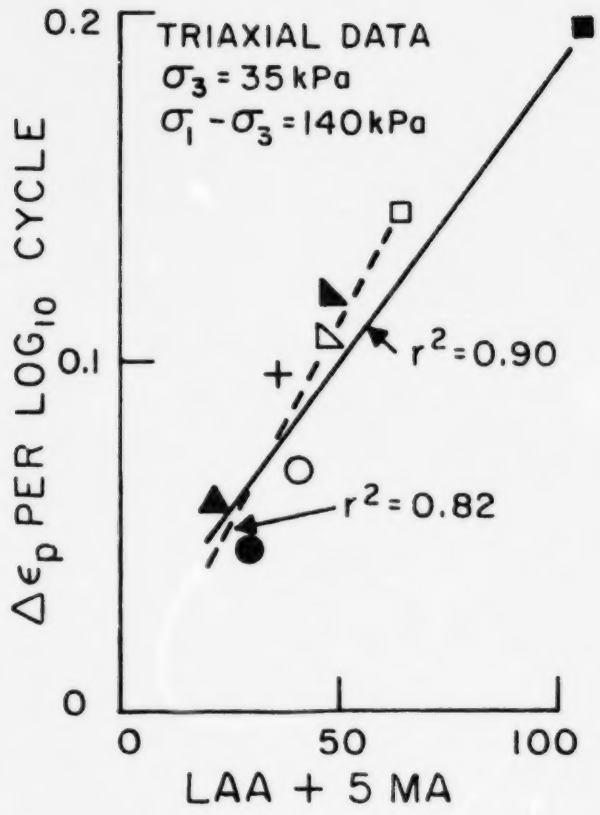


Figure 4/ Plastic Strain per Logarithmic Cycle of Loading versus Aggregate Index Number (Raymond and Diyaljee 1979)

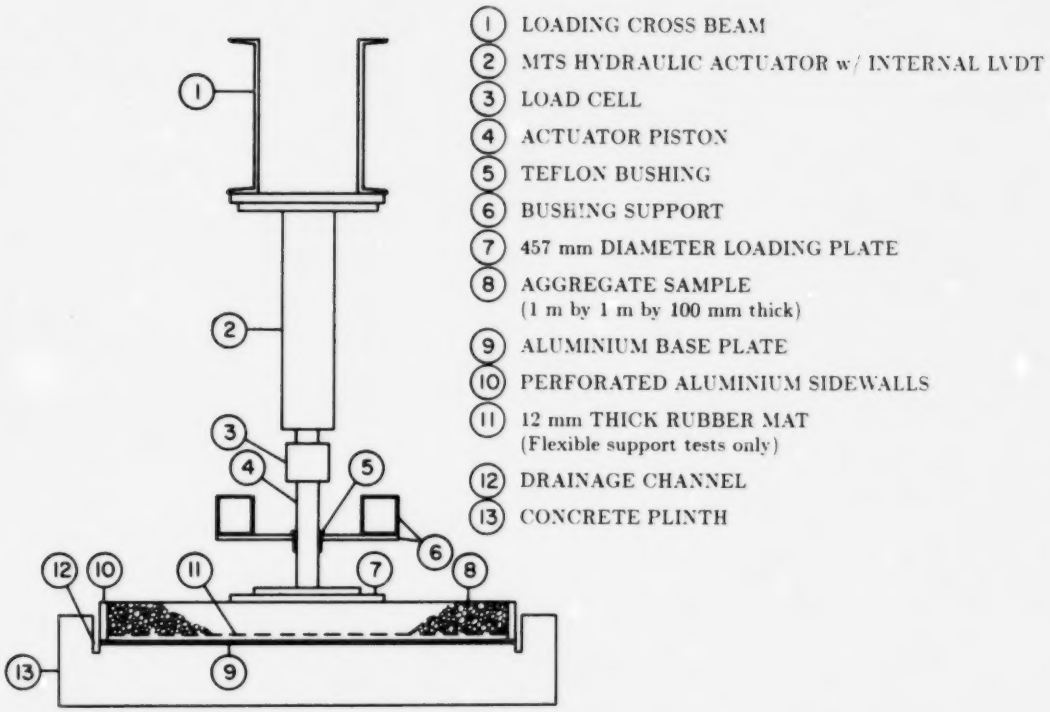

1 LOADING CROSS BEAM
2 MTS HYDRAULIC ACTUATOR w/ INTERNAL LVDT
3 LOAD CELL
4 ACTUATOR PISTON
5 TEFLON BUSHING
6 BUSHING SUPPORT
7 457 mm DIAMETER LOADING PLATE
8 AGGREGATE SAMPLE
(1 m by 1 m by 100 mm thick)
9 ALUMINIUM BASE PLATE
10 PERFORATED ALUMINIUM SIDEWALLS
11 12 mm THICK RUBBER MAT
(Flexible support tests only)
12 DRAINAGE CHANNEL
13 CONCRETE PLINTH

Figure 5/ Plate Bearing Test Apparatus

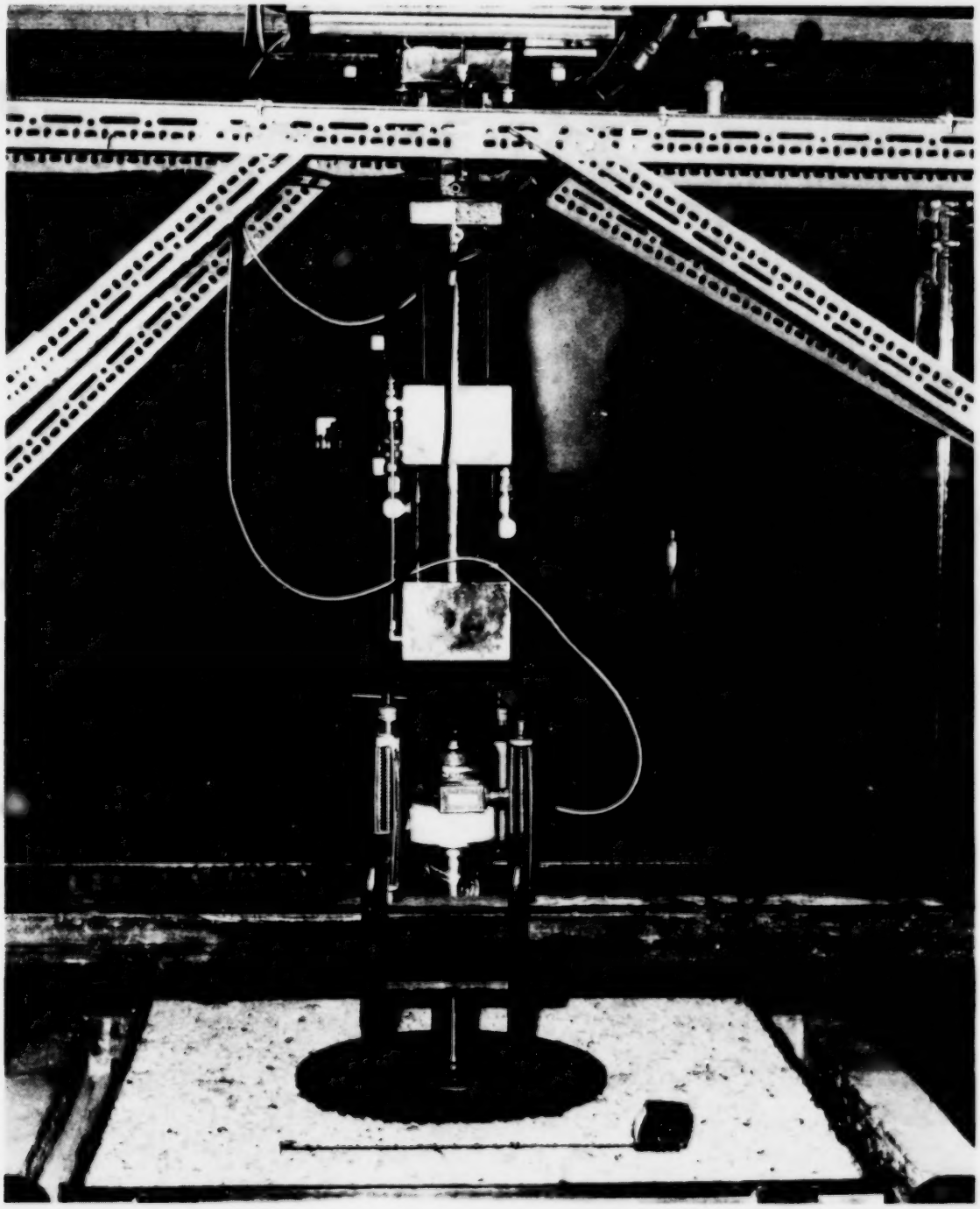


Figure 6/ Overview of Test Facility

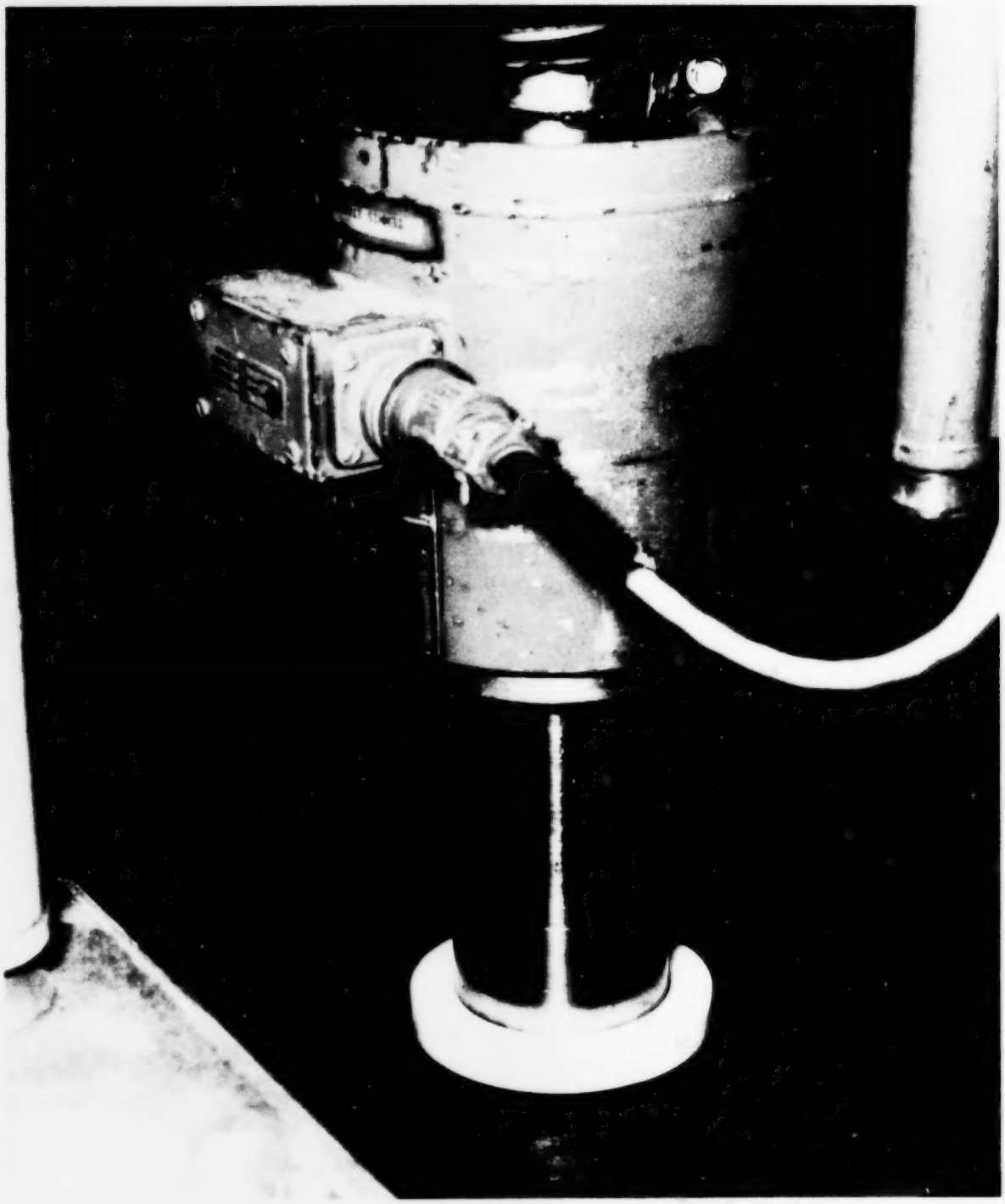
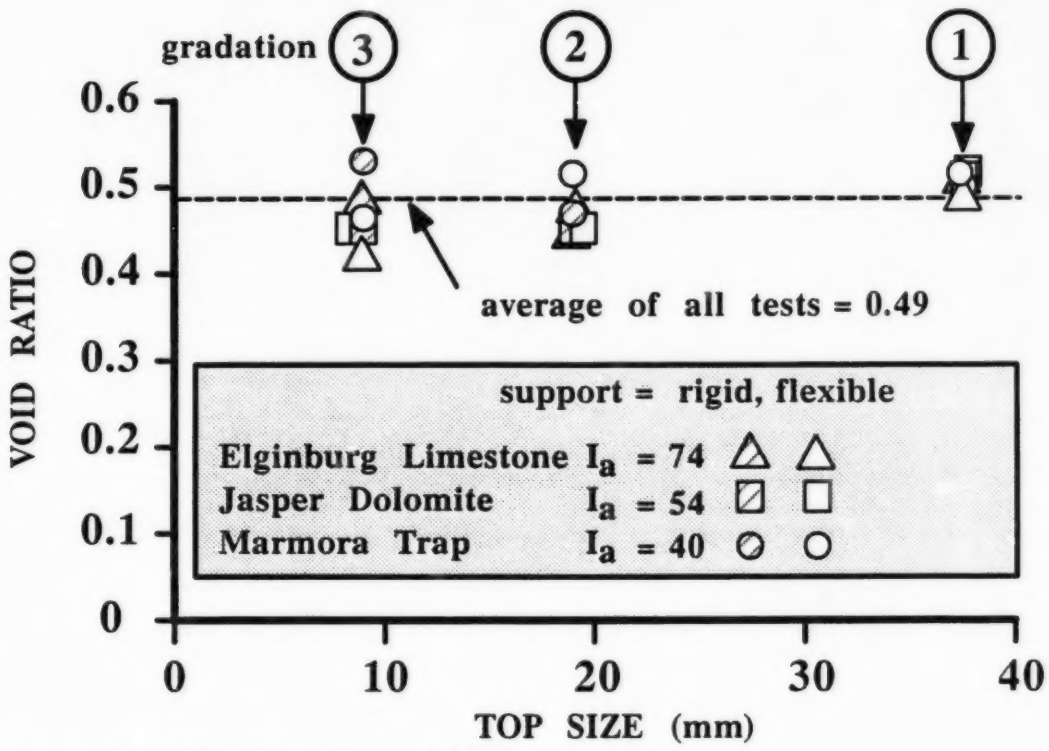
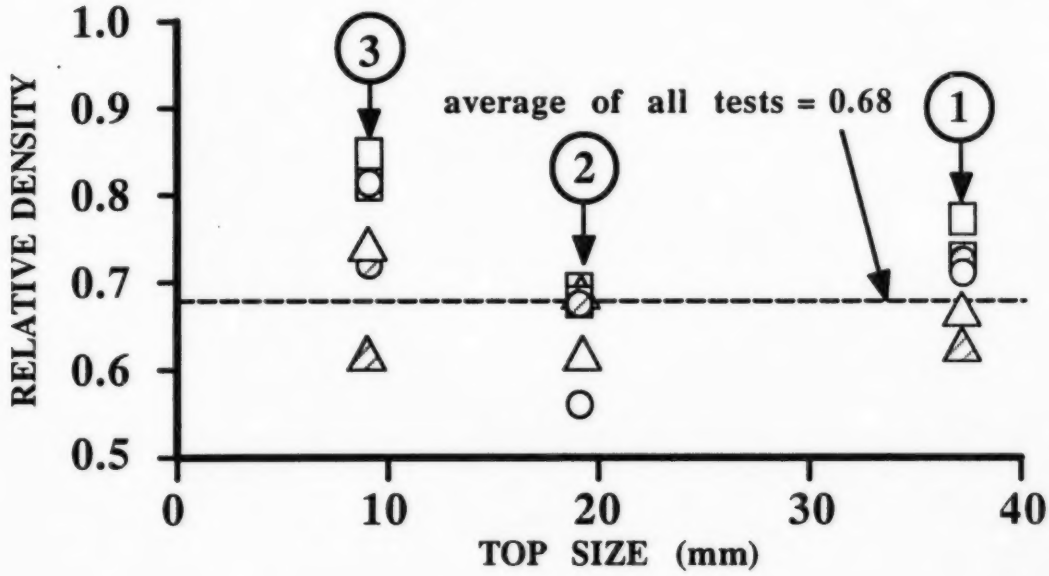


Figure 7/ Teflon Bushing

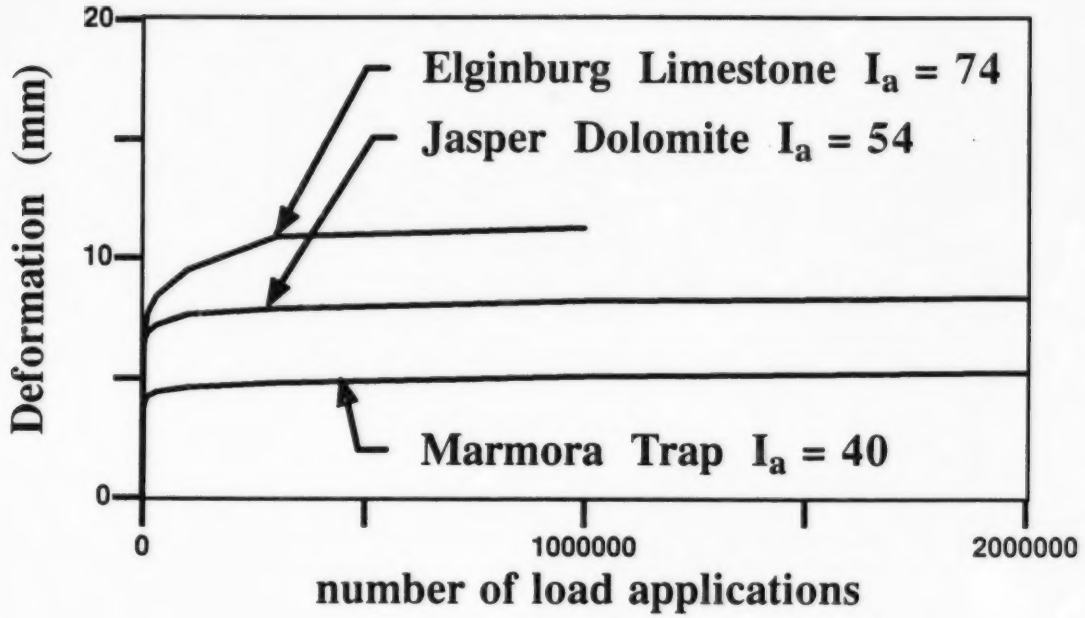


a) void ratio versus top size

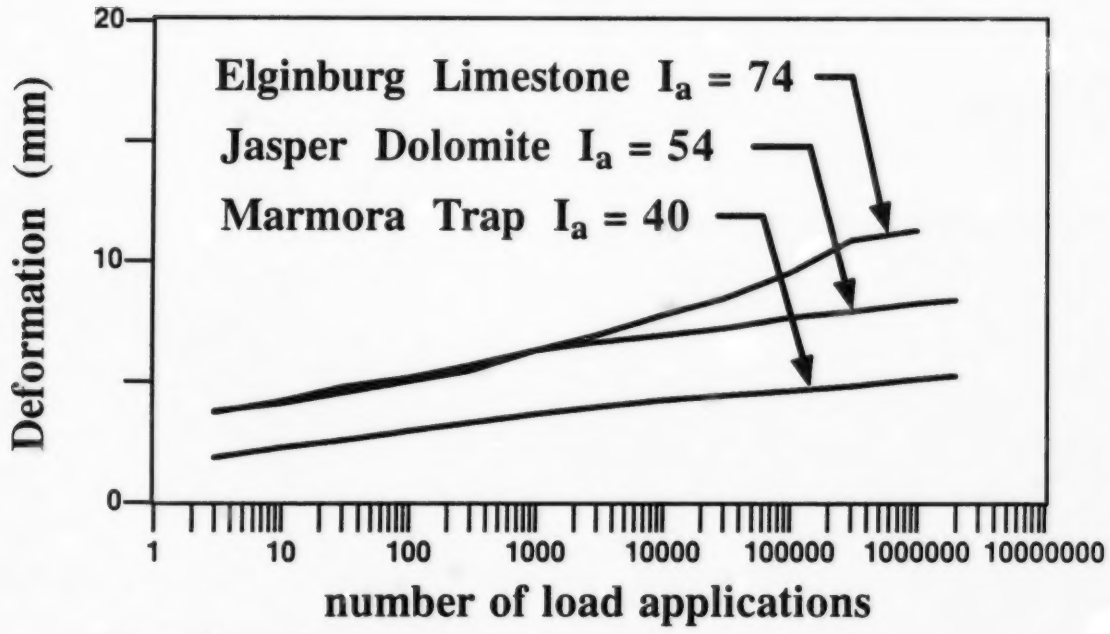


b) relative density versus top size

Figure 8/ Results of Density Tests

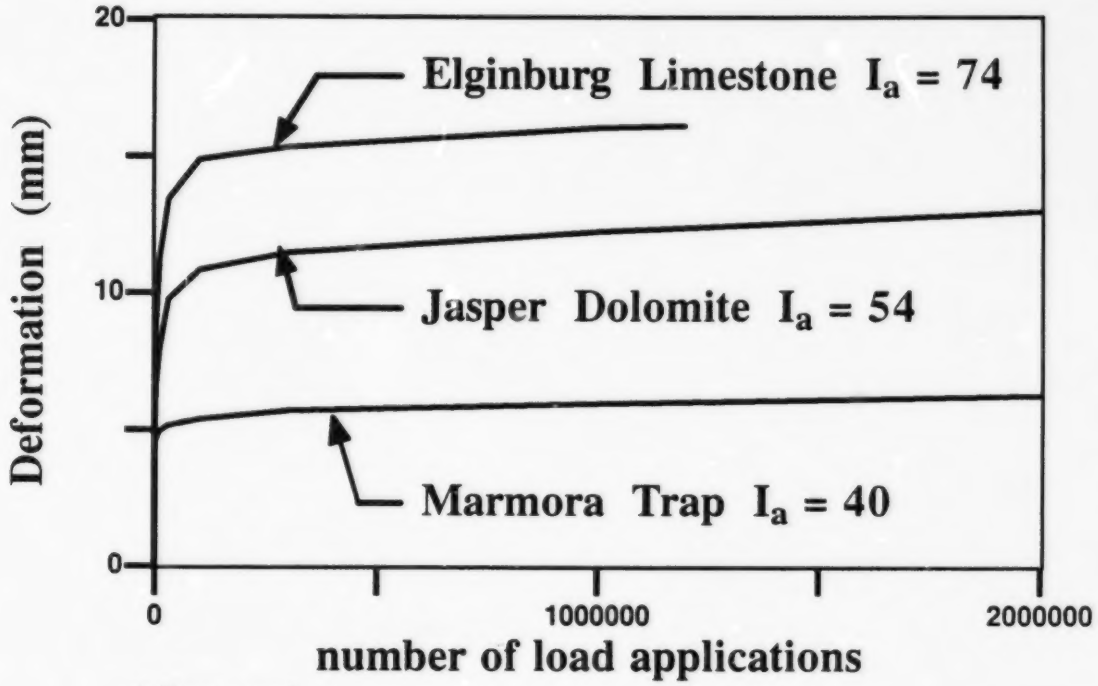


a) linear scale

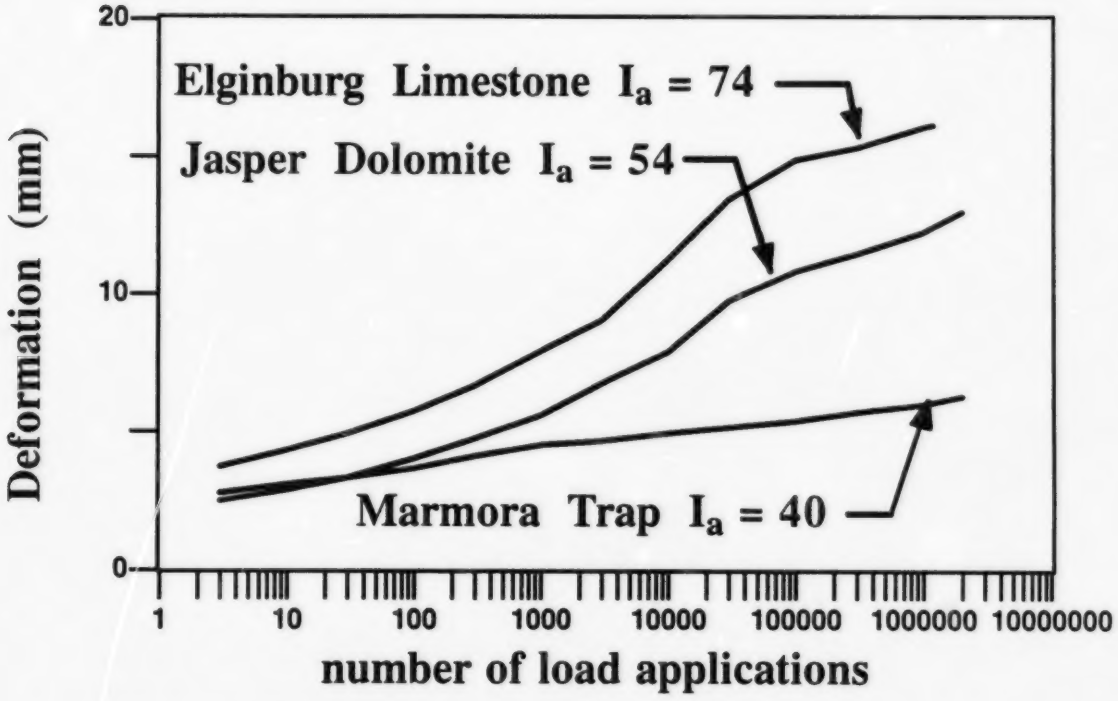


b) semi-logarithmic scale

Figure 9/ Load Deformation Response of Aggregates over RIGID Support
(Gradation 1)

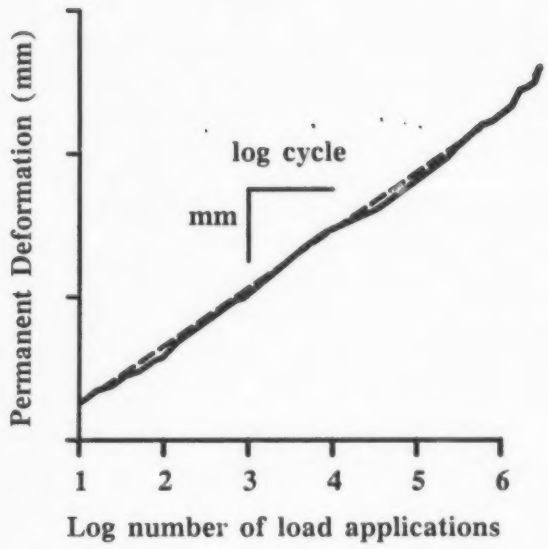


a) linear scale

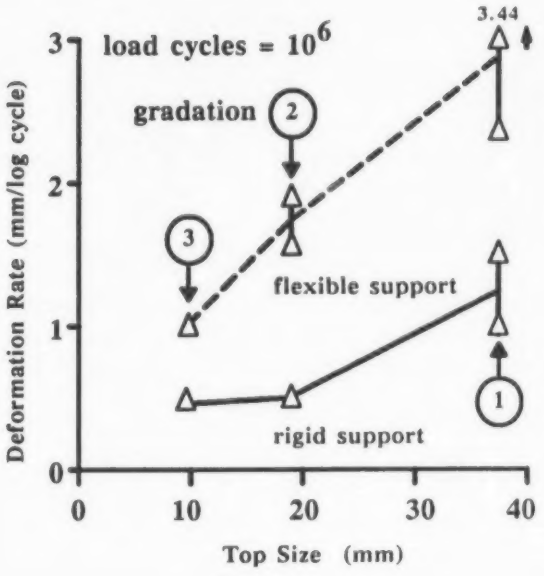


b) semi-logarithmic scale

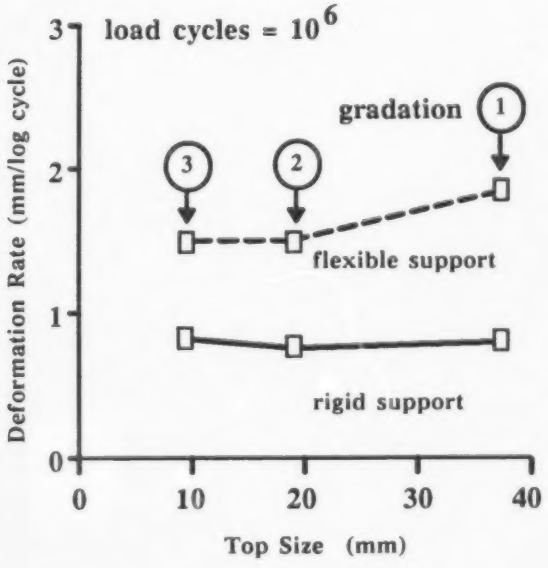
Figure 10/ Load Deformation Response of Aggregates over FLEXIBLE Support (Gradation 1)



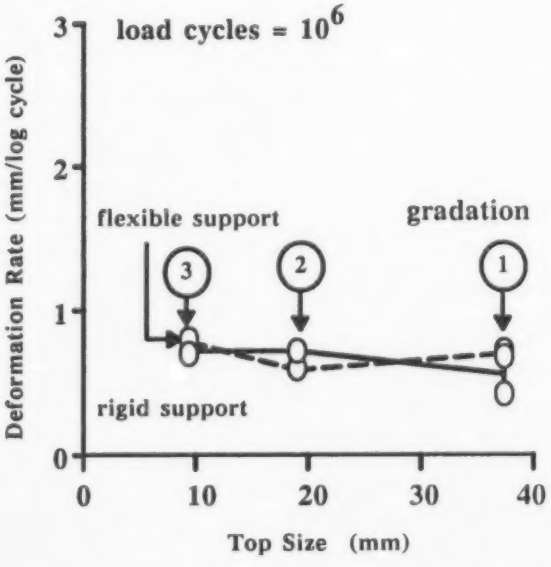
a) definition of deformation rate



b) Elginburg Limestone
(Aggregate Index = 74)

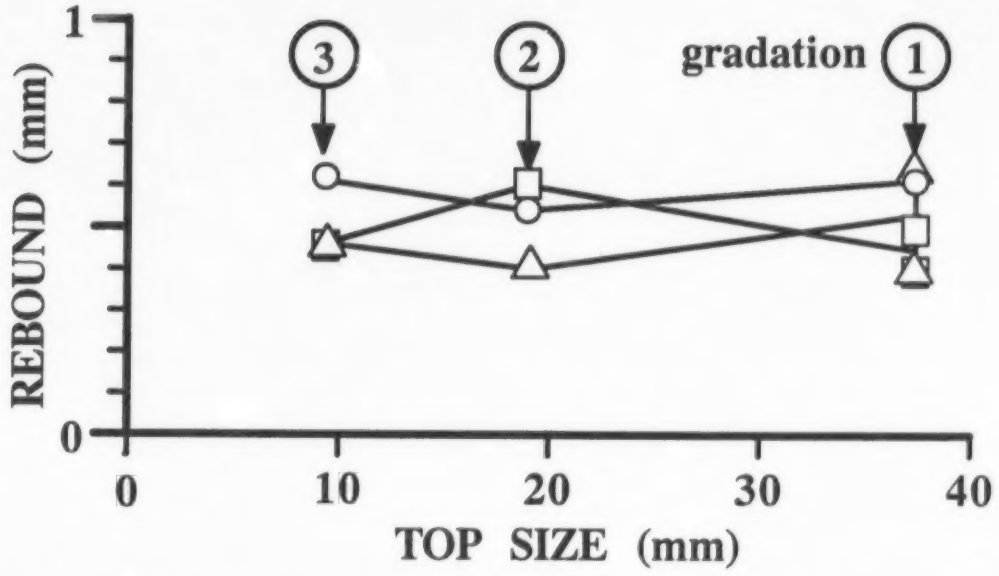


c) Jasper Dolomite
(Aggregate Index = 54)

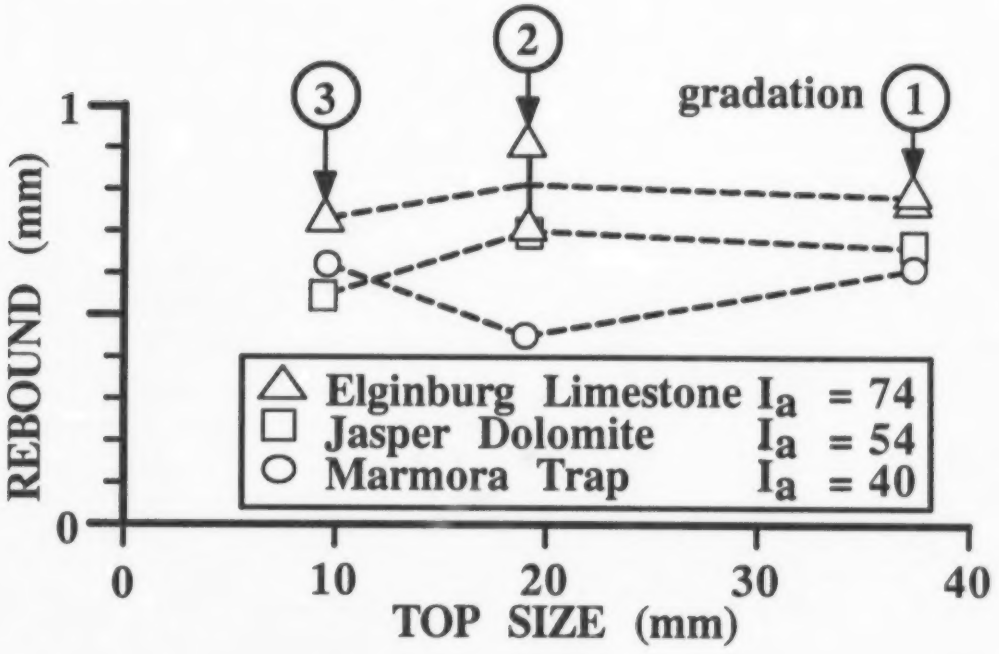


d) Marmora Trap
(Aggregate Index = 40)

Figure 11/ Deformation Rate versus Top Size



a) rigid support



b) flexible support

Figure 12/ Rebound versus Top Size

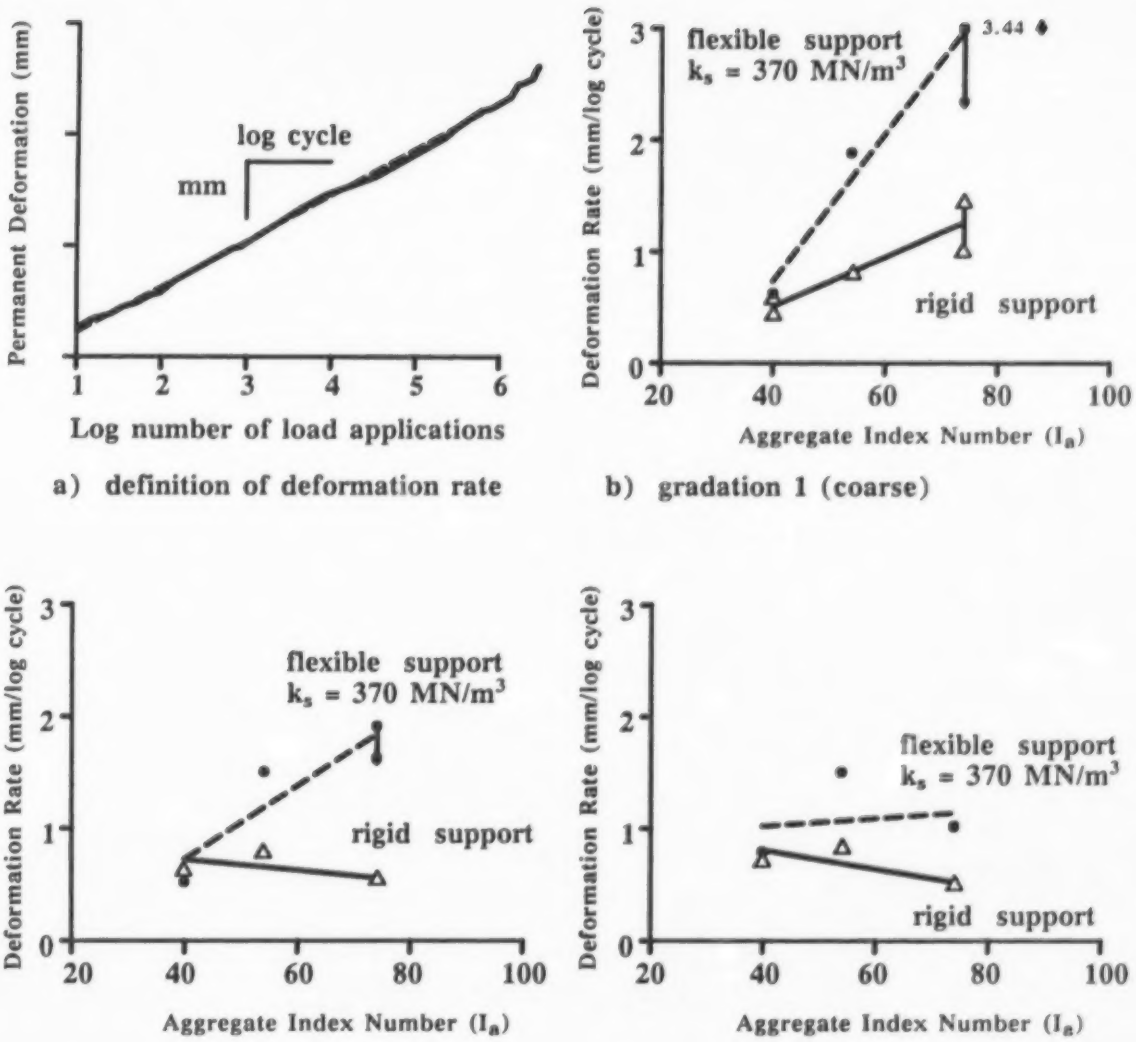


Figure 13/ Deformation Rate versus Aggregate Index Number

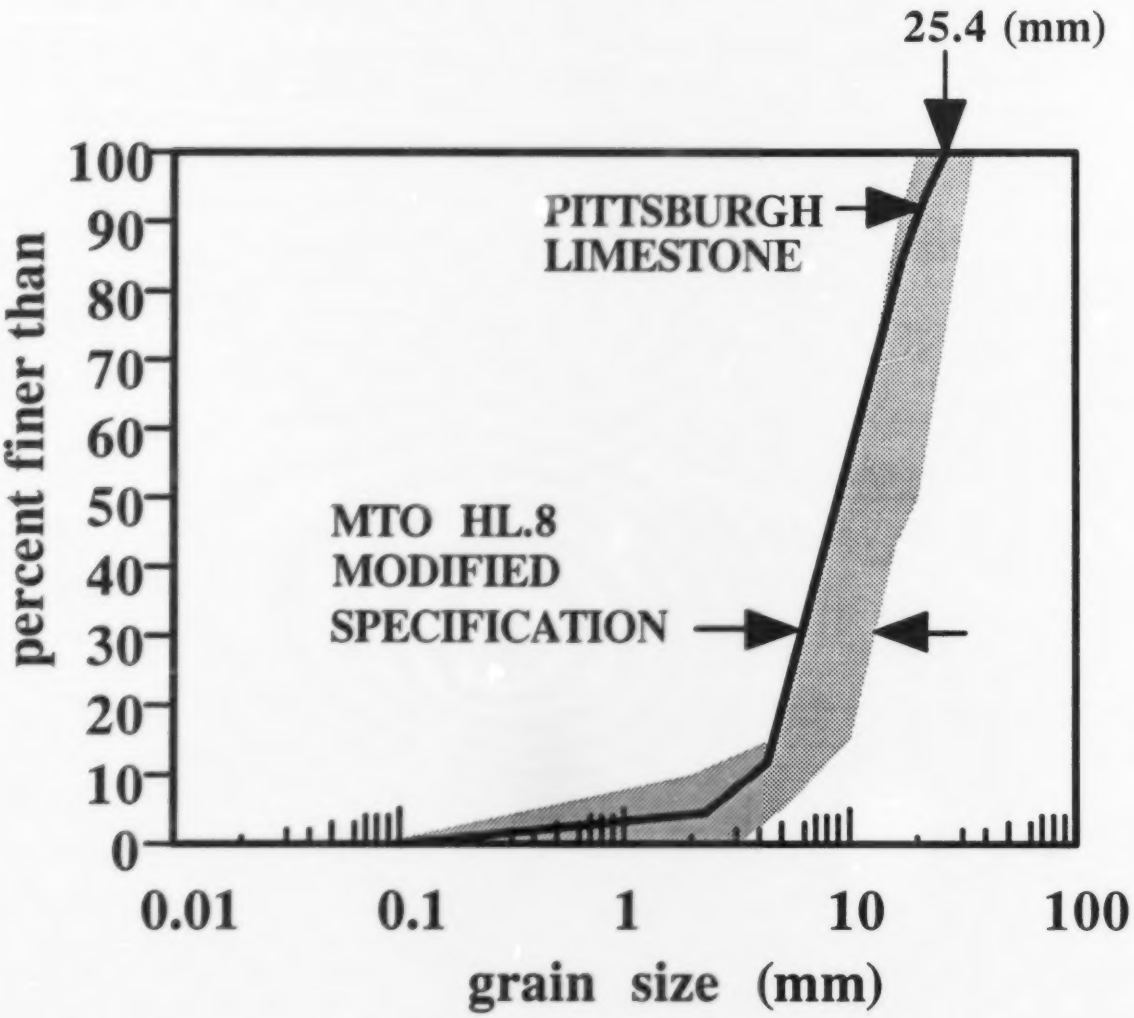


Figure 14/ Grain Size Distribution for Pittsburgh Limestone

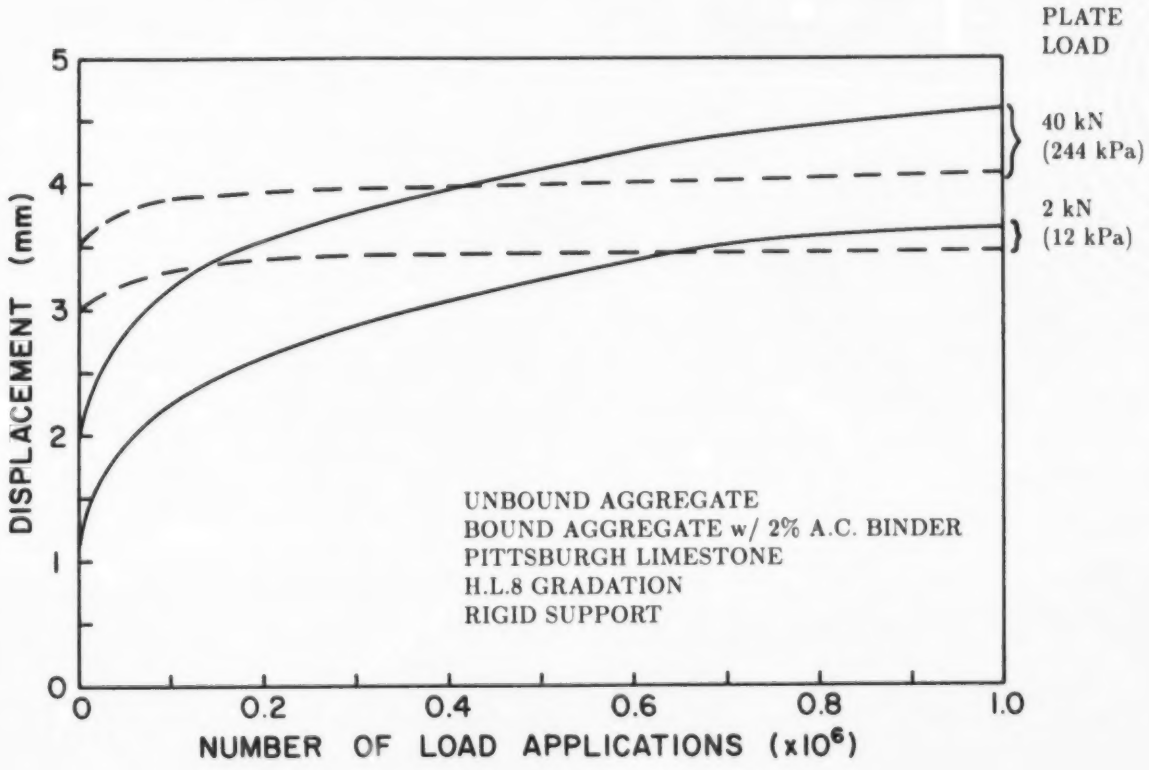


Figure 15/ Load-Deformation Response of Bound and Unbound Open-graded Drainage Layer Samples

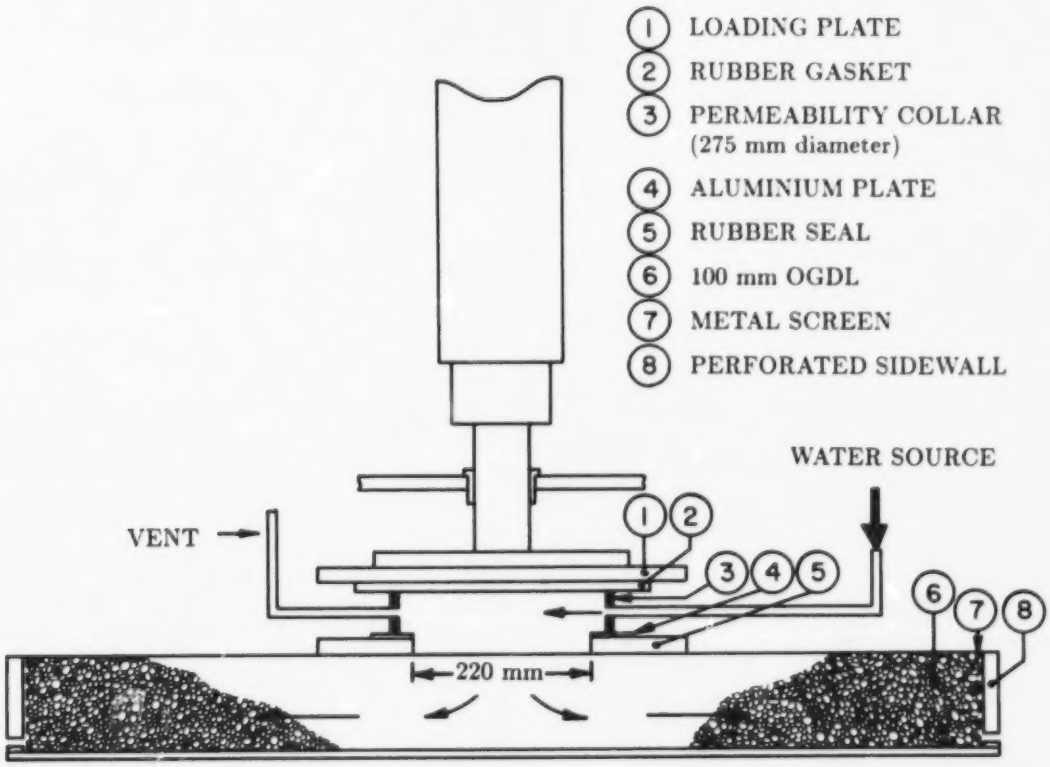


Figure 16/ General Arrangement for In-situ Permeability Testing

APPENDIX A

Current MTO specifications for Open Graded Drainage Layers

The current specifications for Open-Graded Drainage Layers as supplied to us by MTO are as follows:

1) The aggregate shall consist of 100% crushed material.
2) The gradation limits of the aggregate shall correspond to the coarse fraction for HL.4 and HL.8 gradations.
3) The percentage by mass of asphalt cement in the mix shall be 2%.

The gradation limits can be summarized as follows:

MTO Sieve Designation (mm)	Percentage Passing by Dry Mass
26.5	100
19.0	90–100
16.0	65–100
13.2	45–85
9.5	20–55
6.7	8–33
4.75	0–10

In addition, guidelines exist to exclude the use of coated aggregates, flat or elongated particles and crushed aggregate sources with insufficient resistance to fracturing or a high AC stripping potential.

APPENDIX B

Mill Abrasion Test (from CP Rail Specification for Ballast 1984)

The Mill Abrasion Test procedure is as follows:

A representative sample is obtained and sized using current ASTM Methods of Test. From the coarse aggregate, split a representative portion into a sample consisting of 1.5 kg passing the 38.1 mm sieve and retained on the 25.4 mm sieve plus 1.5 kg passing the 25.4 mm sieve and retained on the 19 mm. The sample shall be washed and oven dried in accordance with the Los Angeles Abrasion procedure. The sample will then be placed in a 4.546 litre, 230 mm external diameter porcelain ball mill pot, along with 3 kg of distilled water. The mill pot shall be sealed and rotated at 33 RPM for a total of 10,000 revolutions (five hours). The sample shall then be wash–sieved through a No. 200 sieve and oven–dried before weighing. Mill Abrasion shall be calculated as a percentage loss in weight using the following formula:

$$Mill\ \ Abrasion\ = \frac{loss\ \ in\ \ weight}{original\ \ weight} \times 100$$

Modified Mill Abrasion Test for Small–size Aggregate

The procedure outlined in above was modified in the current investigation for samples of aggregate having particle sizes $\leq$ 19 mm. For these samples a single 3 kg charge was used comprised of particle sizes passing 19 mm and retained on the No. 4 sieve.